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CONTENTS

PAPERS AND DISCUSSIONS

PAGE
The Angular Rotation Method for the Analysis of Multiple Span Frames. *R. A. Freeman* and *A. S. Gelman* . 543

OF GENERAL INTEREST

Example of Design for Wind Bracing in Tall Buildings. *S. C. Jemian* 563
Committees that do not "Committ" 566
Low Water Conditions in the New England District. *H. B. Kinnison* 567
Dinner in Honor of X. Henry Goodnough 572
Proceedings of the Society 574

CONTENTS AND INDEX – Volume XVII, 1930

Contents iii
Index vii

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THE ANGULAR ROTATION METHOD FOR THE ANALYSIS OF MULTIPLE SPAN FRAMES*

By R. A. Freeman† and A. S. Gelman‡

The following method for the analysis of statically indeterminate frames was originated in Europe and developed by Ritter, Suter, Strassner and others. Its application to a wide range of structures is found in the work of Dr. Eng. Ernst Suter, "Die Methode der Festpunkte," published in 1923. Since then the method has gained favor in Europe, and in some parts has become, along with the slope deflection method, one of the standard tools of the designer.

Although radically different in derivation, the angular rotation and the slope deflection methods are based upon the same fundamental principles and assumptions, and when applied to the same system will give identical results. Both methods may be used for the solution of a wide range of structural problems. However, the amount of computation involved may vary considerably. Consequently in most cases one method will prove preferable to the other from the point of view of the practical designer.

The angular rotation method is particularly adaptable to the analysis of multiple-span rigid frames; and a system of this type will be used to illustrate its development and application.

A. General Development

Consider the rigid frame (Fig. 1) loaded in the span *CE* and supported as shown. The broken lines represent the elastic curve of the structure for the given loading. The whole structure may be analyzed as consisting of three sections, namely, frame *ABCD*, the simply supported beam *CE*, and frame *EFGHJ* connected at *C* and *E* by pins (Fig. 2). To compensate for the effect of the pins thus introduced, two equal and opposite moments M_{CE} and two equal and opposite moments

---

* This paper will be presented at a meeting of the Designers Section to be announced later.

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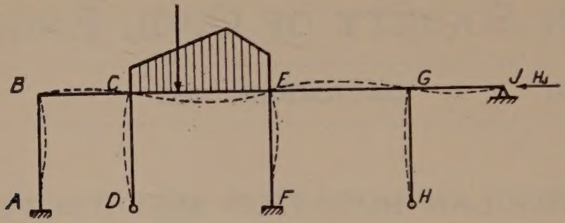


Fig. 1

M_{EC} are applied at *C* and *E*, respectively. It is obvious that these moments equal the corresponding moments in the actual structure.

Assuming that a unit moment applied to the unloaded frame *ABCD* at *C* produces an angular rotation at *C* equal to ϵ_{CE}, the total moment M_{CE} will produce an angular rotation at *C* equal to $M_{CE}\epsilon_{CE}$.

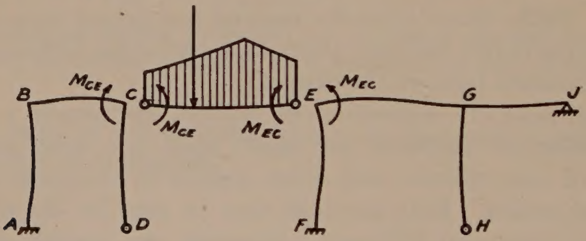


Fig. 2

As there is no break in the elastic curve, this angular rotation must equal the angular rotation of the member *CE* at *C*.

Let α_{CE} be the angular rotation at the end *C* of the simply supported beam *CE* due to a unit moment applied to *CE* at *C* (Fig. 3*a*),

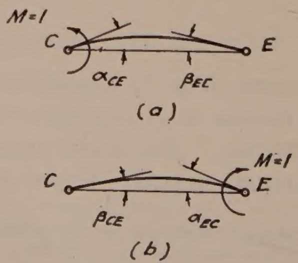
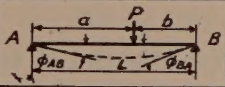
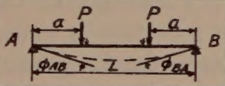
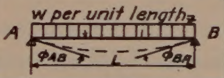
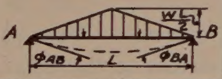


Fig. 3

and let β_{CE} be the angular rotation at the same point due to a unit moment at E (Fig. 3*b*).*

Let ϕ_{CE} be the angular rotation at the end C of the simply supported beam CE produced by the actual loads. (Values of ϕ for a few typical loadings are given in Table I.)

TABLE I

LOADINGS	ϕ_{AB}	ϕ_{BA}
	$\frac{Pab(2b+a)}{6EIL}$	$\frac{Pab(2a+b)}{6EIL}$
 A a P P a B ϕ_{AB} L ϕ_{BA}	$\frac{Pa(L-a)}{2EI}$	$\frac{Pa(L-a)}{2EI}$
 w per unit length A B ϕ_{AB} L ϕ_{BA}	$\frac{wL^3}{24EI}$	$\frac{wL^3}{24EI}$
	$\frac{5wL^4}{384EI}$	$\frac{5wL^4}{384EI}$

Then the net rotation at C of the beam CE equals—

$$\phi_{CE}-M_{CE}\alpha_{CE}-M_{EC}\beta_{CE}$$

and

$$\phi_{CE}-M_{CE}\alpha_{CE}-M_{EC}\beta_{CE}=M_{CE}\epsilon_{CE}$$

Likewise for the point E—

$$\phi_{EC}-M_{EC}\alpha_{EC}-M_{CE}\beta_{EC}=M_{EC}\epsilon_{EC}$$

or

$$\phi_{CE}-M_{CE}\alpha_{CE}-M_{EC}\beta_{CE}-M_{CE}\epsilon_{CE}=0 \qquad (1a)$$

$$\phi_{EC}-M_{EC}\alpha_{EC}-M_{CE}\beta_{EC}-M_{EC}\epsilon_{EC}=0 \qquad (1b)$$

In the development of expression 1*a*, the rotation of joint C consists of three parts; two of these caused by the end moments, and the third (ϕ) produced by the actual loading of the simply supported member. No rotation of the member CE, as a whole, is considered. Consequently, the equation is applicable only when there is no relative displacement of the ends of the members.

---

* It may be easily shown that for any simply supported beam MN with a constant moment of inertia, and a span L, $\alpha_{MN}=\alpha_{NM}=\frac{L}{3\,EI}$ and $\beta_{MN}=\beta_{NM}=\frac{L}{6\,EI}$

For the member BC with the adjoining sections AB on the left and $DC \ldots J$ on the right —

$$\phi_{BC} - M_{BC}\alpha_{BC} - M_{CB}\beta_{BC} - M_{BC}\epsilon_{BC} = 0$$

As there are no loads between B and C —

$$\phi_{BC} = 0$$

and
$$M_{BC}\alpha_{BC} + M_{CB}\beta_{BC} + M_{BC}\epsilon_{BC} = 0$$

or
$$M_{BC} = -M_{CB}\frac{\beta_{BC}}{\alpha_{BC}+\epsilon_{BC}} \qquad (2)$$

The expression (2) cannot be written for the end C, as the rotation of the loaded section $DC \ldots J$ at C is not represented by $M_{CB}\epsilon_{CB}$.

When more than one member of the structure is loaded, the effect of each loaded span or column must be determined separately, and then added together.

The expressions (1a) and (1b) for the loaded member, and (2) for the unloaded members are general, and may be applied to any structure having no relative displacement of joints.

The values of a, ϕ, and ϕ are found directly. The values of ϵ for any member depend upon the elastic properties of the adjoining sections of the structure. For example, to determine ϵ_{CE}, consider section $ABCD$ (Fig. 4). The total moment M_{CE} producing an angular rotation

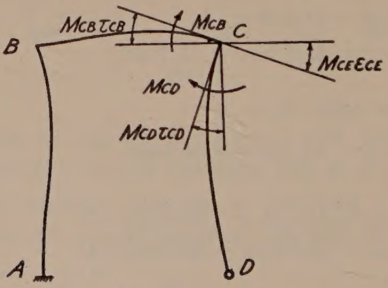


FIG. 4

$M_{CE}\epsilon_{CE}$ may be replaced by the moments M_{CB} and M_{CD} acting on CB and CD, respectively, and producing rotations $M_{CB}\tau_{CB}$ and $M_{CD}\tau_{CD}$ where τ_{CB} is the angular rotation of the end C of the member CB due to a unit moment applied at C on the section ABC of the system. Likewise, τ_{CD} is the rotation produced at C in CD due to a unit moment applied at C.

It is obvious that —

$$M_{CE}\epsilon_{CE} = M_{CB}\tau_{CB} = M_{CD}\tau_{CD}$$

Then —

$$M_{CB} = \frac{M_{CE}\epsilon_{CE}}{\tau_{CB}} \tag{3}$$

$$M_{CD} = \frac{M_{CE}\epsilon_{CE}}{\tau_{CD}}$$

Since —

$$M_{CB} + M_{CD} = M_{CE}$$

$$\frac{M_{CE}\epsilon_{CE}}{\tau_{CB}} + \frac{M_{CE}\epsilon_{CE}}{\tau_{CD}} = M_{CE}$$

or

$$\epsilon_{CE} = \frac{1}{\dfrac{1}{\tau_{CB}} + \dfrac{1}{\tau_{CD}}} \tag{4}$$

To determine the value of τ_{CB}, consider a unit moment applied to ABC at C (Fig. 5).

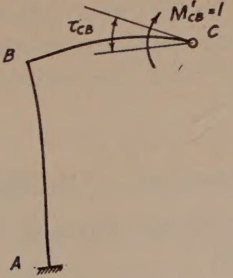


FIG. 5

$$\tau_{CB} = M'_{CB}\alpha_{CB} + M'_{BC}\beta_{CB}$$

As no loads are on ABC, expression (2) may be written —

$$M'_{BC} = -M'_{CB}\frac{\beta_{BC}}{\alpha_{BC} + \epsilon_{BC}}$$

$$= -1\frac{\beta_{BC}}{\alpha_{BC} + \epsilon_{BC}}$$

or

$$\tau_{CB} = (1)\alpha_{CB} - (1)\frac{\beta_{BC}}{\alpha_{BC} + \epsilon_{BC}}\beta_{CB} \tag{5}$$

This expression includes ϵ_{BC}, the value of which is as yet unknown. But from (4) —

$$\epsilon_{BC} = \frac{1}{\frac{1}{\tau_{BA}}} = \tau_{BA}$$

In turn (5) may be rewritten for τ_{BA}:

$$\tau_{BA} = \alpha_{BA} - \frac{\beta_{AB}\beta_{BA}}{\alpha_{AB} + \epsilon_{AB}}$$

Here ϵ_{AB} represents the angular rotation of the support A. Since the base A is fixed, $\epsilon_{AB} = 0$.

Similarly for τ_{CD} —

$$\tau_{CD} = \alpha_{CD} - \frac{\beta_{DC}\beta_{CD}}{\alpha_{DC} + \epsilon_{DC}}$$

and $\epsilon_{DC} = \infty$ (pinned support)

On the basis of $\epsilon_{AB} = 0$ and $\epsilon_{DC} = \infty$, the values of τ_{CB}, τ_{CD}, and ϵ_{CE} may be found. Likewise the τ's and ϵ's for the entire structure are determined. Substituting in (1*a*) and (1*b*) computed values for α, β, ϕ and ϵ, the moments at the ends of the loaded member are obtained. Then, using (2) and (3) in turn, the moments throughout the system are computed.

B. Computation of Moments — No Displacement of Joints

As an example, the structure shown in Fig. 6, with members 1, 2, 3, 4 and 5, will be analyzed.

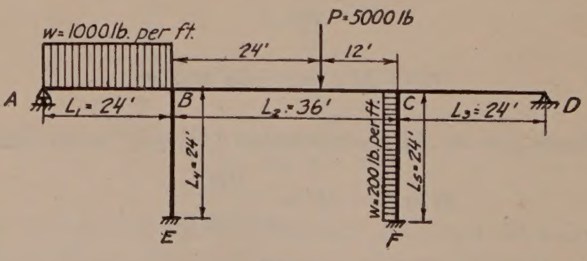


FIG. 6

Let —

$$I_1 = 2I_0 \qquad I_3 = 2I_0 \qquad I_5 = I_0$$
$$I_2 = 3I_0 \qquad I_4 = I_0$$

Assuming each member to have a constant moment of inertia throughout its length —

$$\alpha_{AB} = \alpha_{BA} = \alpha_1 = \frac{L_1}{3EI_1}, \text{ etc.}$$

and $$\beta_{AB} = \beta_{BA} = \beta_1 = \frac{L_1}{6EI_1}, \text{ etc.}$$

Values of α and β thus computed for all members are given in Table II:

TABLE II

MEMBER	L	$3\,EI$	$6\,EI$	α	β		τ	ϵ	ϕ
1 . . .	24	$6\,EI_0$	$12\,EI_0$	$\frac{4}{EI_0}$	$\frac{2}{EI_0}$	AB	—	a	$\frac{288{,}000}{EI_0}$
						BA	$\frac{4}{EI_0}$	$\frac{2.16}{EI_0}$	$\frac{288\,000}{EI_0}$
2 . . .	36	$9\,EI_0$	$18\,EI_0$	$\frac{4}{EI_0}$	$\frac{2}{EI_0}$	BC	$\frac{3.37}{EI_0}$	$\frac{2.4}{EI_0}$	$\frac{106{,}670}{EI_0}$
						CB	$\frac{3.37}{EI_0}$	$\frac{2.4}{EI_0}$	$\frac{133{,}330}{EI_0}$
3 . . .	24	$6\,EI_0$	$12\,EI_0$	$\frac{4}{EI_0}$	$\frac{2}{EI_0}$	CD	$\frac{4}{EI_0}$	$\frac{2.16}{EI_0}$	—
						DC	—	a	—
4 . . .	24	$3\,EI_0$	$6\,EI_0$	$\frac{8}{EI_0}$	$\frac{4}{EI_0}$	BE	$\frac{6}{EI_0}$	$\frac{1.83}{EI_0}$	—
						EB	—	0	—
5 . . .	24	$3\,EI_0$	$6\,EI_0$	$\frac{8}{EI_0}$	$\frac{4}{EI_0}$	CF	$\frac{6}{EI_0}$	$\frac{1.83}{EI_0}$	$\frac{115{,}200}{EI_0}$
						FC	—	0	$\frac{115{,}200}{EI_0}$

The values of τ and ϵ must next be determined.

For the member AB with the pinned support A, $\epsilon_{AB} = \infty$.

From (5) —

$$\tau_{BA} = \alpha_{BA} - \frac{\beta_{AB}\beta_{BA}}{\alpha_{AB} + \epsilon_{AB}} = \alpha_1 - \frac{\beta_1^2}{\alpha_1 + \epsilon_{AB}}$$

Substituting numerical values from Table II —

$$\tau_{BA} = \frac{4}{EI_0} - \frac{\left(\frac{2}{EI_0}\right)^2}{\frac{4}{EI_0} + \infty} = \frac{4}{EI_0}$$

For column EB with the fixed base E —

$$\epsilon_{EB} = 0$$

and $$\tau_{BE} = \alpha_4 - \frac{\beta_4^2}{\alpha_4 + \epsilon_{EB}} = \frac{8}{EI_0} - \frac{\left(\frac{4}{EI_0}\right)^2}{\frac{8}{EI_0} + 0} = \frac{6}{EI_0}$$

From (4) —

$$\epsilon BC = \frac{1}{\frac{1}{\tau BA}+\frac{1}{\tau BE}} = \frac{1}{\frac{EI_0}{4}+\frac{EI_0}{6}} = \frac{2.4}{EI_0}$$

$$\tau CB = a_2 - \frac{\beta_2^{\,2}}{a_2+\epsilon BC} = \frac{4}{EI_0} - \frac{\left(\frac{2}{EI_0}\right)^2}{\frac{4}{EI_0}+\frac{2.4}{EI_0}} = \frac{3.37}{EI_0}$$

For column FC —

$$\epsilon FC = 0$$

and
$$\tau CF = a_5 - \frac{\beta_5^{\,2}}{a_5+\epsilon FC} = \frac{8}{EI_0} - \frac{\left(\frac{4}{EI_0}\right)^2}{\frac{8}{EI_0}+0} = \frac{6}{EI_0}$$

$$\epsilon CD = \frac{1}{\frac{1}{\tau CB}+\frac{1}{\tau CF}} = \frac{1}{\frac{EI_0}{3.37}+\frac{EI_0}{6}} = \frac{2.16}{EI_0}$$

For support D —

$$\epsilon DC = \infty$$

and
$$\tau CD = a_3 - \frac{\beta_3^{\,2}}{a_3+\epsilon DC} = \frac{4}{EI_0} - \frac{\left(\frac{2}{EI_0}\right)^2}{\frac{4}{EI_0}+\infty} = \frac{4}{EI_0}$$

$$\epsilon CB = \frac{1}{\frac{1}{\tau CD}+\frac{1}{\tau CF}} = \frac{1}{\frac{EI_0}{4}+\frac{EI_0}{6}} = \frac{2.4}{EI_0}$$

$$\tau BC = a_2 - \frac{\beta_2^{\,2}}{a_2+\epsilon CB} = \frac{4}{EI_0} - \frac{\left(\frac{2}{EI_0}\right)^2}{\frac{4}{EI_0}+\frac{2.4}{EI_0}} = \frac{3.37}{EI_0}$$

$$\epsilon BA = \frac{1}{\frac{1}{\tau BC}+\frac{1}{\tau BE}} = \frac{1}{\frac{EI_0}{3.37}+\frac{EI_0}{6}} = \frac{2.16}{EI_0}$$

$$\epsilon BE = \frac{1}{\frac{1}{\tau BA}+\frac{1}{\tau BC}} = \frac{1}{\frac{EI_0}{4}+\frac{EI_0}{3.37}} = \frac{1.83}{EI_0}$$

$$\epsilon CF = \frac{1}{\frac{1}{\tau CB}+\frac{1}{\tau CD}} = \frac{1}{\frac{EI_0}{3.37}+\frac{EI_0}{4}} = \frac{1.83}{EI_0}$$

It may be seen that $\tau BA = \tau CD$, $\epsilon BC = \epsilon CB$, $\epsilon BE = \epsilon CF$, etc., as a result of the symmetry of the structure.

All the inherent elastic properties involved in the moment equations (1) (2) and (3) are now known. Before the moments may be determined, it remains to compute values of ϕ, which depend on loading.

These values as computed from formulas in Table I are given in Table II, together with values of α, β, ϵ, and τ.

Consider, first, the effect of loads located in span AB. Values of moments at the ends of the loaded member AB are determined from equations (1a) and (1b).

$$\phi_{AB} - M_{AB}\alpha_{AB} - M_{BA}\beta_{AB} - M_{AB}\epsilon_{AB} = 0$$
$$\phi_{BA} - M_{BA}\alpha_{BA} - M_{AB}\beta_{BA} - M_{BA}\epsilon_{BA} = 0$$

Since $M_{AB}=0$, M_{BA} is found directly from the second equation. The first equation cannot be used, since $M_{AB}\epsilon_{AB}$, which equals $0 \times \infty$, is an indeterminate quantity.

Thus —

$$M_{BA}\alpha_{BA} + M_{BA}\epsilon_{BA} - \phi_{BA} = 0$$

or $$M_{BA} = \phi_{BA}\frac{1}{\alpha_1 + \epsilon_{BA}}$$

Substituting values of α_1, ϵ_{BA}, and ϕ_{BA} —

$$M_{BA} = \frac{288{,}000}{EI_0}\,\frac{1}{\frac{4}{EI_0}+\frac{2.16}{EI_0}} = 46{,}750$$

From (3) —

$$M_{BC} = M_{BA}\frac{\epsilon_{BA}}{\tau_{BC}} = 46{,}750\frac{\frac{2.16}{EI_0}}{\frac{3.37}{EI_0}} = 29{,}950$$

and $$M_{BE} = M_{BA}\frac{\epsilon_{BA}}{\tau_{BE}} = 46{,}750\frac{\frac{2.16}{EI_0}}{\frac{3.67}{EI_0}} = 16{,}800$$

Using expression (2) —

$$M_{EB} = M_{BE}\frac{\beta_4}{\alpha_4 + \epsilon_{EB}} = 16{,}800\frac{\frac{4}{EI_0}}{\frac{8}{EI_0}+0} = 8{,}400$$

and $$M_{CB} = M_{BC}\frac{\beta_2}{\alpha_2 + \epsilon_{CB}} = 29{,}950\frac{\frac{2}{EI_0}}{\frac{4}{EI_0}+\frac{2.4}{EI_0}} = 9{,}350$$

Again from formula (3) —

$$M_{CD} = \frac{M_{CB}\epsilon_{CB}}{\tau_{CD}} = 9{,}350\frac{\frac{2.4}{EI_0}}{\frac{4}{EI_0}} = 5{,}610$$

and $$M_{CF} = \frac{M_{CB}\epsilon_{CB}}{\tau_{CF}} = 9{,}350\frac{\frac{2.4}{EI_0}}{\frac{6}{EI_0}} = 3{,}740$$

From (2) —

$$M_{FC}=M_{CF}\frac{\beta_5}{\alpha_5+\epsilon_{FC}}=3{,}740\,\frac{\frac{4}{EI_0}}{\frac{8}{EI_0}+0}=1{,}870$$

and $M_{DC}=0$ for pinned support at D

The directions of the moments computed above are indicated in Fig. 7*a*, and are determined on the basis of the following obvious properties of rigid frames (if no relative displacement of joints occurs):

A. The moments at the ends of a loaded member resist the actual loading.

B. The moments at the ends of an unloaded member are in the same direction (either both clockwise or both counterclockwise).

C. At any joint, the moment at the end of the loaded member, or the member leading towards the loaded span, is resisted by all the other moments at that joint. A similar sketch is drawn for every loading investigated. For convenience of tabulation, clockwise moments are considered positive; counterclockwise, negative.

In the above computations, the values of EI_0 cancel each other in all expressions, and consequently the moments are obtained in the units used for length and loads, regardless of the units used for E and I.

Consider the effect of loading in span BC. Substituting values of α, β, ϵ, and ϕ (EI_0 omitted), equations (1*a*) and (1*b*) become —

$$106{,}670-4\,M_{BC}-2\,M_{CB}-2.4\,M_{BC}=0$$
$$133{,}330-4\,M_{CB}-2\,M_{BC}-2.4\,M_{CB}=0$$

Solving —

$$M_{CB}=17{,}330$$
$$M_{BC}=11{,}250$$

These moments are distributed throughout the system by formulas (2) and (3).

$$M_{BA}=M_{BC}\frac{\epsilon_{BC}}{\tau_{BA}}=11{,}250\frac{2.4}{4.0}=6{,}750$$

$$M_{BE}=M_{BC}\frac{\epsilon_{BC}}{\tau_{BE}}=11{,}250\frac{2.4}{6.0}=4{,}500$$

$$M_{AB}=0$$

$$M_{EB}=M_{BE}\frac{\beta_4}{\alpha_4+\epsilon_{EB}}=4{,}500\frac{4}{8+0}=2{,}250$$

$$M_{CD}=M_{CB}\frac{\epsilon_{CB}}{\tau_{CD}}=17{,}330\frac{2.4}{4.0}=10{,}400$$

$$M_{CF}=M_{CB}\frac{\epsilon_{CB}}{\tau_{CF}}=17{,}330\frac{2.4}{6.0}=6{,}930$$

$$M_{DC}=0$$

$$M_{FC}=M_{CF}\frac{\beta_5}{\alpha_5+\epsilon_{FC}}=6{,}930\frac{4}{8+0}=3{,}470$$

For directions of moments see Fig. *7b*.

Treating the loading on column *CF* similarly —

$$115{,}200-8\ M_{CF}-4\ M_{FC}-1.83\ M_{CF}=0$$
$$115{,}200-8\ M_{FC}-4\ M_{CF}-0\ M_{FC}=0$$
$$M_{CF}=7{,}360$$
$$M_{FC}=10{,}720$$

$$M_{CB}=M_{CF}\frac{\epsilon_{CF}}{\tau_{CB}}=7{,}360\frac{1.83}{3.37}=4{,}000$$

$$M_{CD}=M_{CF}\frac{\epsilon_{CF}}{\tau_{CD}}=7{,}360\frac{1.83}{4.0}=3{,}360$$

$$M_{BC}=M_{CB}\frac{\beta_2}{\alpha_2+\epsilon_{BC}}=4{,}000\frac{2}{4+2.4}=1{,}250$$

$$M_{BA}=M_{BC}\frac{\epsilon_{BC}}{\tau_{BA}}=1{,}250\frac{2.4}{4.0}=750$$

$$M_{BE}=M_{BC}\frac{\epsilon_{BC}}{\tau_{BE}}=1{,}250\frac{2.4}{6.0}=500$$

$$M_{EB}=M_{BE}\frac{\beta_4}{\alpha_4+\epsilon_{EB}}=500\frac{4}{8+0}=250$$

$$M_{AB}=0$$

Directions of moments are indicated on Fig. 7*c*.

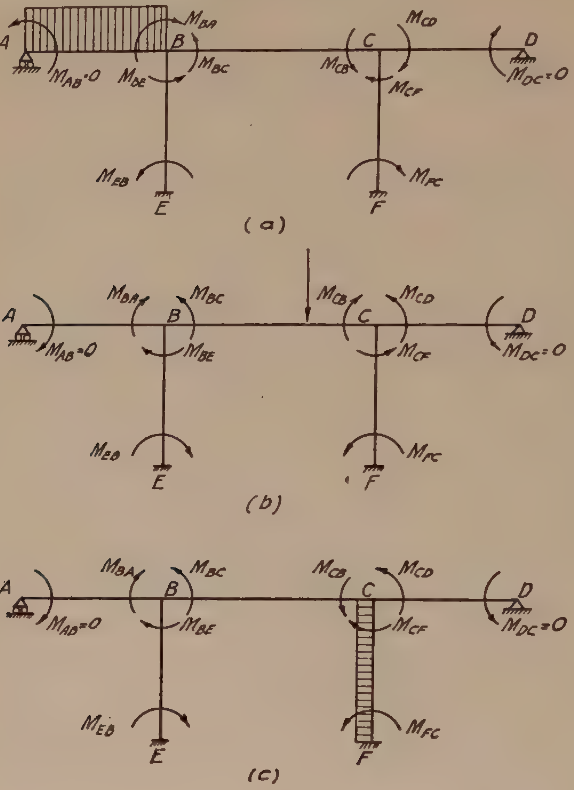


FIG. 7

The total moments are obtained as the algebraic sum of the independent effects of each loading, and are shown in Table III.

TABLE III

MOMENT	Effect of Loading on *AB*	Effect of Loading on *BC*	Effect of Loading on *CF*	Total
MAB	0	0	0	0
MBA	+46,750	+6,750	+750	+54,250
MBC	−29,950	−11,250	−1,250	−42,450
MBE	−16,800	+4,500	+500	−11,800
MEB	−8,400	+2,250	+250	−5,900
MCB	−9,350	+17,330	−4,000	+3,980
MCF	+3,740	−6,930	+7,360	+4,170
MCD	+5,610	−10,400	−3,360	−8,150
MFC	+1,870	−3,470	−10,720	−12,320
MDC	0	0	0	0

The most unfavorable loading condition for any point in the structure is readily determined by analyzing the moment tabulation.

The moments due to any additional loading may be calculated independently and added to those already obtained.

C. Effect of Horizontal Displacement

In the general development it was assumed that no horizontal displacement of joints occurred. If rollers are provided at J (Fig. 1), a horizontal movement will result, and moments and shears in the system will be changed. With the support J fixed horizontally, a horizontal reaction, H_J, exists at that point. The introduction of rollers at J is equivalent to the removal of the reaction, H_J. Consequently the structure will be distorted as though acted upon by a force at J equal and opposite to H_J. Total moments for the system with rollers at J equal the algebraic sum of the moments produced by actual loads in the system horizontally supported at J, and those due to the force H_J applied on the system with rollers.

Assume the structure deflected a given distance Δ, as shown in Fig. 8*a*, by a force H_0 applied at the point J. The moments produced by H_0 in this system are equal to those caused by the displacement of supports A, D, F, and H, as shown in Fig. 8*b*, as the elastic curves in both cases are identical. If the moments introduced by the displacement Δ of base A only (Fig. 8*c*), and those introduced by the displacement Δ of base D only (Fig. 8*d*), etc., are determined, their algebraic sum will equal the moments in the system shown in Fig. 8*a*.

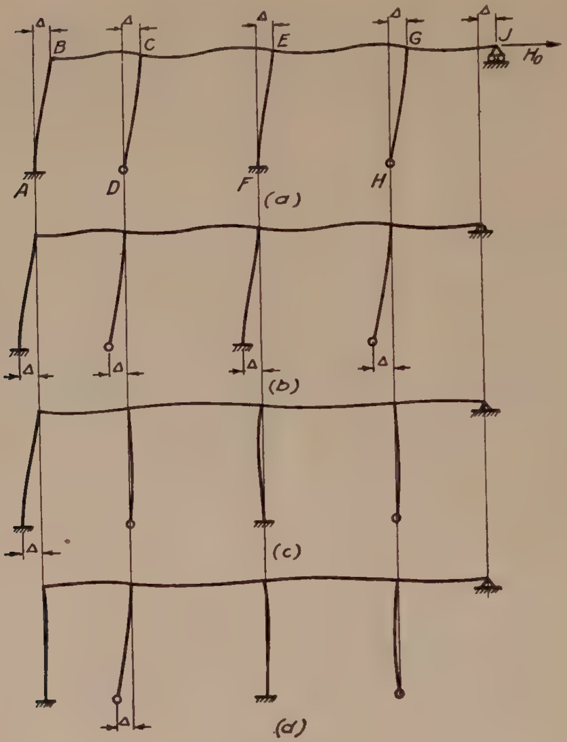


FIG. 8

Consider, first, the effect of the displacement Δ of the base A (Fig. 8c).

Applying the method used in the general development, and assuming the column AB pinned at A and B with the end moments M'_{AB} and M'_{BA}, for the end B—

$$\frac{\Delta}{L_{AB}} - M'_{BA}\alpha_{BA} + M'_{AB}\beta_{BA} - M'_{BA}\epsilon_{BA} = 0 \qquad (6a)$$

and for A—

$$\frac{\Delta}{L_{AB}} - M'_{AB}\alpha_{AB} + M'_{BA}\beta_{AB} - M'_{AB}\epsilon_{AB} = 0 \qquad (6b)$$

These expressions are similar to (1a) and (1b), with $\frac{\Delta}{L}$ corresponding to ϕ. In this case, the rotation $M'_{AB}\beta_{BA}$ (6a) is in the same direction as $\frac{\Delta}{L_{AB}}$, and is used with the same sign.

The section $BCD \ldots J$ of the system (Fig. 8c) is identical with the corresponding section of the original system (Fig. 1). Consequently, ϵ_{BA} is the same for both cases (ϵ_{AB} depends on base conditions alone), and the moments M_{BA} and M_{AB} are determined directly from (6*a*) and (6*b*). Furthermore, the effect of M_{BA} throughout the section $BCD \ldots J$ is found by the application of formulas (2) and (3), using the values of ϵ and τ already determined for the original system (Fig. 1).

The moments produced by the displacement of each other base are found similarly and added to the effect of the displacement of base A.

The totals thus obtained are the moments due to the force H_0 acting on the system shown in Fig. 8*a*. The magnitude of H_0 is readily determined from these moments. To obtain the effect of H_J upon the structure it is necessary to multiply the above moments by the ratio $\frac{H_J}{H_0}$.

For convenience of reference, the computations of moments for the horizontally supported structure will be called "Case I," and those for the effect of horizontal displacement, "Case II."

D. COMPUTATION OF MOMENTS — HORIZONTAL DISPLACEMENT OF JOINTS

The system (Fig. 6) previously solved will be analyzed with rollers at both supports A and D. In this new system, assume that the horizontal force H_0 produces a deflection $\Delta = \frac{1{,}000\, L_4}{EI_0}$.

The moments due to the displacement of base E only (Fig. 9) are determined as follows: —

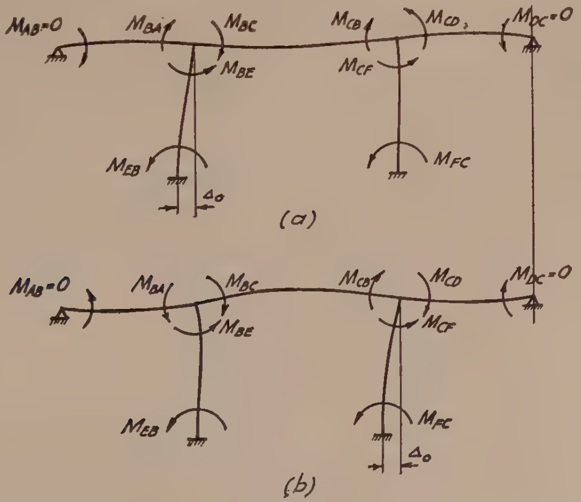


FIG. 9

From (*6a*) and (*6b*) —

$$\frac{1,000}{EI_0}-\frac{8}{EI_0}M'_{BE}+\frac{4}{EI_0}M'_{EB}-\frac{1.83}{EI_0}M'_{BE}=0$$

$$\frac{1,000}{EI_0}-\frac{8}{EI_0}M'_{EB}+\frac{4}{EI_0}M'_{BE}-0\,M'_{EB}=0$$

Solving —

$$M'_{BE}=191.5$$

$$M'_{EB}=220.7$$

$$M'_{BA}=M'_{BE}\frac{\epsilon_{BE}}{\tau_{BA}}=191.5\frac{1.83}{4.0}=87.5$$

$$M'_{BC}=M'_{BE}\frac{\epsilon_{BE}}{\tau_{BC}}=191.5\frac{1.83}{3.37}=104.0$$

$$M'_{CB}=M'_{BC}\frac{\beta_2}{a_2+\epsilon_{CB}}=\frac{104.0\times 2}{4+2.4}=32.5$$

$$M'_{CD}=M'_{CB}\frac{\epsilon_{CB}}{\tau_{CD}}=32.5\frac{2.4}{4.0}=19.5$$

$$M'_{CF}=M'_{CB}\frac{\epsilon_{CB}}{\tau_{CF}}=32.5\frac{2.4}{6.0}=13.0$$

$$M'_{FC}=M'_{CF}\frac{\beta_5}{a_5+\epsilon_{FC}}=13.0\frac{4}{8+0}=6.5$$

The directions of all moments are indicated in Fig. 9*a*.

The moments produced by the displacement $\triangle=\frac{1,000\,L_4}{EI_0}$ at base *F* with *E* stationary may be similarly computed. Due to the symmetry of the system these moments may be written directly while their directions are shown in Fig. 9*b*.

The moments due to the force H_0 are listed in Table IV. The force H_0 equals the algebraic sum of H'_E and H'_F, which are the reactions at the bases *E* and *F* due to the displacement $\triangle$.

$$H'_E=-\frac{M'_{BE}+M'_{EB}}{L_4}=\frac{204.5+227.2}{24}=18.0$$

$$H'_F=-\frac{M'_{CF}+M'_{FC}}{L_5}=\frac{204.5+227.2}{24}=18.0$$

and $H_0=H'_E+H'_F=36.0$

From Case I:

$$H_E = -\frac{M_{BE}+M_{EB}}{L_4} = \frac{11{,}800+5{,}900}{24} = 737$$

$$H_F = 200\frac{L_5}{2} - \frac{M_{CF}+M_{FC}}{L_5} = 2{,}400 - \frac{4{,}170-12{,}320}{24} = 2{,}740$$

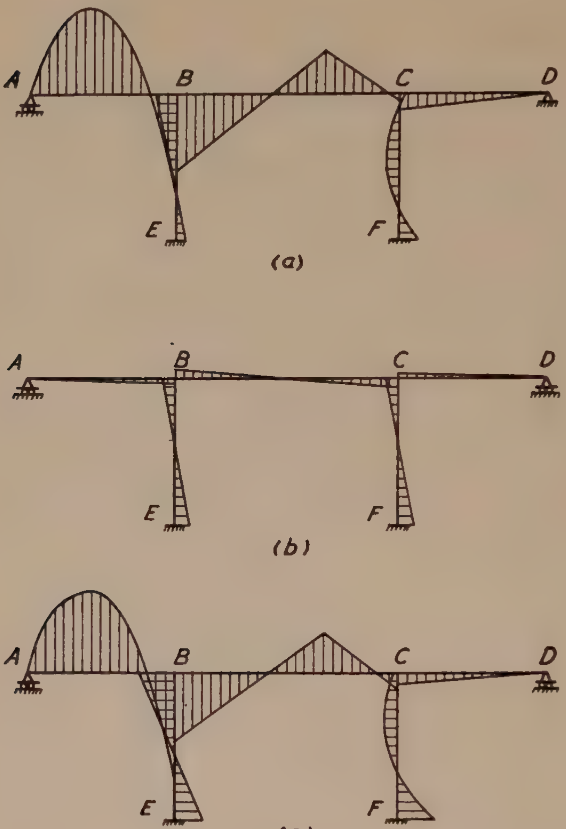
$$H_D = 200\times 24 - H_E - H_F = 1{,}323$$

To obtain the effect of H_D, the moments due to H_0 are multiplied by the ratio $\frac{H_D}{H_0} = \frac{1{,}323}{36} = 36.8$, and the results are given in Table IV.

TABLE IV

MOMENT	Effect of Displacement of *E*	Effect of Displacement of *F*	Total (Effect of) H_0	Effect of H_D
M'_{AB}	0	0	0	0
M'_{BA}	+87.5	−19.5	+68.0	+2,500
M'_{BC}	+104.0	+32.5	+136.5	+5,020
M'_{BE}	−191.5	−13.0	−204.5	−7,530
M'_{EB}	−220.7	−6.5	−227.2	−8,350
M'_{CB}	+32.5	+104.0	+136.5	+5,020
M'_{CF}	−13.0	−191.5	−204.5	−7,530
M'_{CD}	−19.5	+87.5	+68.0	+2,500
M'_{FC}	−6.5	−220.7	−227.2	−8,350
M'_{DC}	0	0	0	0

The total moments produced by the actual loading on the system with rollers at *A* and *D* are determined as the algebraic sum of the moments of Case I and of Case II. The moments of Case I and Case II as well as total moments are listed in Table V, and corresponding moment diagrams are shown in Fig. 10 with all moments plotted on the compression side of the members.



(a)

(b)

(c)

FIG. 10

Figs. 10*a* and 10*b* represent the moments of Case I and Case II, respectively, while the total moments are shown in Fig. 10*c*. All three diagrams are plotted to the same scale.

TABLE V

MOMENT	Case I	Case II	Total Moment
M_{AB}	0	0	0
M_{BA}	+54,250	+2,500	+56,750
M_{BC}	−42,450	+5,020	−37,430
M_{BE}	−11,800	−7,530	−19,330
M_{EB}	−5,900	−8,350	−14,250
M_{CB}	+3,980	+5,020	+9,000
M_{CF}	+4,170	−7,530	−3,360
M_{CD}	−8,150	+2,500	−5,650
M_{FC}	−12,320	−8,350	−20,670
M_{DC}	0	0	0

E. SUMMARY

The two numerical cases just completed illustrate the application of the angular rotation method to multiple span rigid frames.

Thus, it is first necessary to compute the values of α, β, τ, and ϵ, which are elastic properties of the system and are independent of type and location of loads. If the system has the possibility of horizontal displacement, a temporary horizontal support must be provided. The effect throughout the structure of each loaded span is computed separately by the method of Case I, and the moments for any number of loaded spans are obtained by summation. On the basis of these moments, the values of horizontal reactions are computed. The temporary horizontal support is then removed, and the horizontal reaction just computed for this support is considered as an unbalanced force on the original structure. The moments due to this force are computed by the method of Case II, and are added to the moments already found. If the system has no horizontal displacement of joints, the moments of Case I are final, and Case II is eliminated.

It is often desirable to consider supports neither pinned nor fully fixed, but in an intermediate condition of restraint depending on base design and soil characteristics. In such cases values of ϵ for the supports are somewhere between zero and infinity, and cannot be readily estimated. However, the point of contraflexure in an unloaded column is somewhere between the one-third point ($\epsilon = 0$) and the base ($\epsilon = \infty$), and its position for any base condition can be estimated with a fair degree of certainty. This position determines the ratio of the moment at the base to the moment at the top of the column when no loads are located on that column. This ratio has been shown to be: $\dfrac{\beta}{\alpha + \epsilon BASE}$. Thus, a value of $\epsilon BASE$ corresponding to the assumed position of the point of contraflexure is determined for a given α and β, and should be used consistently throughout the analysis of the structure.

In the application of the angular rotation method to actual problems, the following features stand out:

1. When a structure has been once analyzed for any system of loads, it may be analyzed for any other system of loads without much additional work.
2. In the process of computation of moments, the effect of each loading appears separately. This separation is useful in various ways, and especially as it simplifies the determination of critical loading conditions.

3. The effect of partial restraint at supports may be carried throughout the system consistently, and does not introduce any new complications.

4. The mathematics is simple and direct. Small differences between large numbers are never involved, and consequently the slide rule will give sufficient precision in the majority of practical cases.

5. The computation of moments is for magnitude only, and is not complicated by the question of signs. The directions of moments are found independently by use of simple and safe rules.

The solution of multiple span frames has been illustrated. The method may also be used to great advantage in the case of continuous beams over numerous supports with various loadings and moments of inertia (ϵ becomes corresponding τ).

The adaptation of the principles of this method to more complicated systems, such as frames with closed contours, curved members, multiple-story frames, etc., may be found in the extensive treatise on the subject by Dr. Suter.

OF GENERAL INTEREST

EXAMPLE OF DESIGN FOR WIND BRACING IN TALL BUILDINGS

By S. C. Jemian[*]

The paper presented by Mr. Edward Smulski, on "Wind Bracing," at the meeting of the Boston Society of Civil Engineers, held on September 24, 1930, was of considerable interest.

It is believed, also, that a numerical example will provide for a better understanding of the method which I referred to in my discussion at that meeting, as well as offering proof of several of my statements concerning the distribution of horizontal loads between column lines.

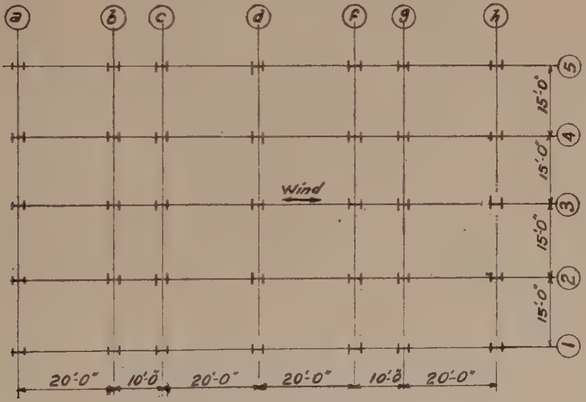


FIG. 1.—PLAN

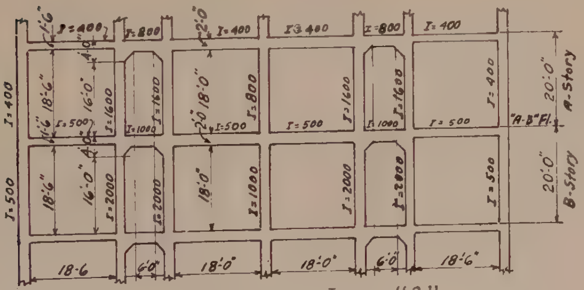


FIG. 2.—COLUMN LINES "3"

Two stories of a typical building are chosen for this purpose. The plan and elevations are shown by Figs. 1, 2, 3 and 4. Moments of inertia and clear heights of columns and clear lengths of beams are given on the elevations. Table 1 shows how to proceed with the distribution of the total wind load, above the story under consideration. The total wind load above story "A" is 84,000 pounds, and above story "B," 108,000 pounds. Table 2 is prepared for a comparison of the horizontal shears carried by each column line. At story "A" each exterior column line carries 21.8 per cent, column lines "2" and "4" each carry 17.4 per cent, and column line "3" carries 21.6 per cent. At story "B" column lines "1" and

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[*] With Stone & Webster Engineering Corporation, Boston.

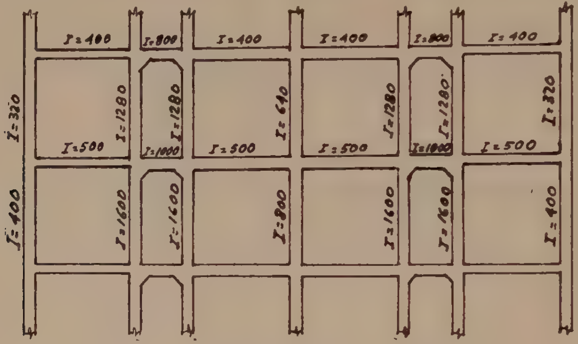

FIG. 3. — COLUMN LINES "2" AND "4"

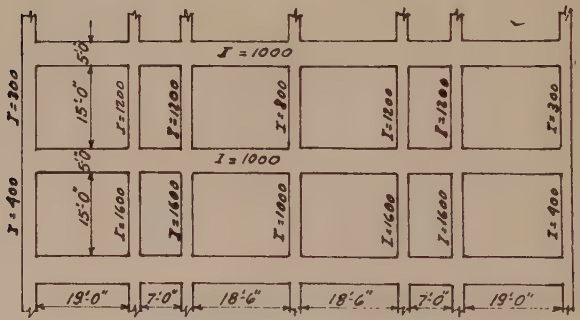

FIG. 4. — COLUMN LINES "1" AND "5"

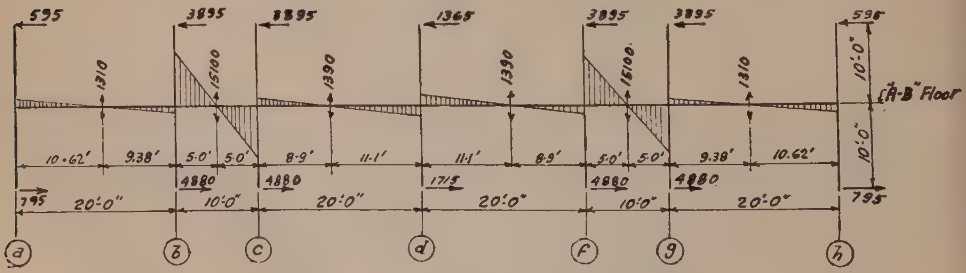

FIG. 5. — SHEAR AT COLUMN LINES "3"

"5" each carry 22.5 per cent, column lines "2" and "4" each carry 21.0 per cent, and column line "3" carries 21.0 per cent of the total wind load above the story in consideration. With the methods in common use each one of the exterior column lines "1" and "5" is figured to carry 12.5 per cent, and each one of the interior column lines to carry 25 per cent, of the load.

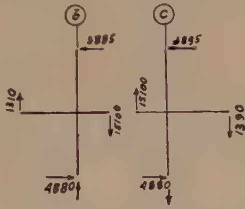

FIG. 6

TABLE 1.

Col.	A-Story: Total Shear= 84000*					B-Story: Total Shear= 108000*				
	$\frac{h}{2}$ ft.	$(\frac{h}{2})^3$	I in.4	$\frac{I}{(\frac{h}{2})^3}$	$S_1=S\frac{\frac{I}{(h/2)^3}}{\sum\frac{I}{(h/2)^3}}$	$\frac{h}{2}$	$(\frac{h}{2})^3$	I	$\frac{I}{(\frac{h}{2})^3}$	$S_1=S\frac{\frac{I}{(h/2)^3}}{\sum\frac{I}{(h/2)^3}}$
a1	7.5	421.8	300	0.712	890			400	0.948	1190*
a2	9.25	791.4	320	0.404	505			400	0.505	630
a3	9.25	do	400	0.556	595			500	0.631	795
a4	9.25	do	320	0.404	505			400	0.505	630
a5	7.5	421.8	300	0.712	890			400	0.948	1190
b1	7.5	do	1200	2.845	3545			1600	3.795	4740
b2	8.0	512.0	1280	2.500	3125			1600	3.125	3920
b3	8.0	do	1600	3.125	3895			2000	3.905	4880
b4	8.0	do	1280	2.500	3125			1600	3.125	3920
b5	7.5	421.8	1200	2.845	3545			1600	3.795	4740
c1	7.5	do	1200	2.845	3545			1600	3.795	4740
c2	8.0	512.0	1280	2.500	3125			1600	3.125	3920
c3	8.0	do	1600	3.125	3895			2000	3.905	4880
c4	8.0	do	1280	2.500	3125			1600	3.125	3920
c5	7.5	421.8	1200	2.845	3545			1600	3.795	4740
d1	7.5	do	800	1.900	2370			1000	2.370	2960
d2	9.0	729.0	640	0.878	1095	Same as A-Story	Same as A-Story	800	1.097	1370
d3	9.0	do	800	1.097	1365			1000	1.371	1715
d4	9.0	do	640	0.878	1095			800	1.097	1370
d5	7.5	421.8	800	1.900	2375			1000	2.370	2960
F1	7.5	do	1200	2.845	3545			1600	3.795	4740
F2	8.0	512.0	1280	2.500	3125			1600	3.125	3920
F3	8.0	do	1600	3.125	3895			2000	3.905	4880
F4	8.0	do	1280	2.500	3125			1600	3.125	3920
F5	7.5	421.8	1200	2.845	3545			1600	3.795	4740
g1	7.5	do	1200	2.845	3545			1600	3.795	4740
g2	8.0	512.0	1280	2.500	3125			1600	3.125	3920
g3	8.0	do	1600	3.125	3895			2000	3.905	4880
g4	8.0	do	1280	2.500	3125			1600	3.125	3920
g5	7.5	421.8	1200	2.845	3545			1600	3.795	4740
h1	7.5	do	300	0.712	890			400	0.948	1190
h2	9.25	791.4	320	0.404	505			400	0.505	630
h3	9.25	do	400	0.556	595			500	0.631	795
h4	9.25	do	320	0.404	505			400	0.505	630
h5	7.5	421.8	300	0.712	890			400	0.948	1190
Totals				67.489	84000				86.359	108000

The rest of the work can be completed by tables or graphically. Table 3 shows the method of determining the moments in the beams.

Having obtained the joint moments of the beams, the points of zero moment and consequently the vertical shears are determined easily, as shown by Fig. 5.

TABLE 2.

Col.	A. Story			B. Story		
	1&5	2&4	3	1&5	2&4	3
a	890	505	595	1190	630	795
b	3546	3126	3895	4740	3920	4880
c	3545	3126	3895	4740	3920	4880
d	2370	1095	1365	2960	1370	1715
f	3545	3125	3895	4740	3920	4880
g	3545	3125	3895	4740	3920	4880
h	890	505	595	1190	630	795
Total	18330	14605	18135	24300	18810	22825
%→	21.8	17.4	21.6	22.5	17.0	21.0

Columns "B" and "C" are separated with their respective shears (see Fig. 6).

The reaction at "B" is a compression and equal to 15,100 − 1,310 = 13,790 pounds, and the reaction of "C" is an uplift and equal to 15,100 − 1,390 = 13,710 pounds.

TABLE 3.

Floor	Joint	Beam at left			Beam at right			$\Sigma\frac{I}{l}$	Col. Mt. above	Col. Mt. below	ΣM	Mom't at left	Mom't at right
		I	l	$\frac{I}{l}$	I	l	$\frac{I}{l}$						
Floor A-B	a3	—	—	—	500	18.5	27.0	27.0	5950	7950	13900	—	13900
	b3	500	18.5	27.0	1000	6.0	166.7	193.7	38950	48800	87750	12250	75500
	c3	1000	6.0	166.7	500	18.0	27.7	193.7	38950	48800	87750	75500	12250
	d3	500	18.0	27.7	500	18.0	27.7	55.4	13650	17150	30800	15400	15400
	f3	500	18.0	27.7	1000	6.0	166.7	193.7	38950	48800	87750	12250	75500
	g3	1000	6.0	166.7	500	18.5	27.0	1937	38950	48800	87750	75500	12250
	h3	500	18.5	27.0	—	—		27.0	5950	7950	13900	13900	—

This last investigation shows clearly that the common ways of finding axial loads in columns are far from the actuality. One of these methods assumes that interior columns would not have any axial load, and the other one assumes that axial loads of columns will follow the straight line stress theory, or, in other words, that on one side of the center line of the framing all columns will have compression, and on the other side uplift.

The present example shows clearly that on the same side of the center line there can be compression as well as uplift.

In conclusion it can be stated that the loads instead of following our arbitrary assumptions follow the laws of the "strength of materials."

Committees That Do Not "Committ"

By a Committeeman

I

It is not a pleasant task to start an anvil chorus, but it seems a necessary one. Our committees apparently are merely a sort of dormant honors-list, nothing more. That should not be so, for committees are appointed to do work, not merely for the sake of filling the inside back cover of the JOURNAL with lists of names.

During the last fiscal year the Library Committee was apparently the only one that undertook a program of work to carry through. The accomplishment of that committee was the result of the energy of its chairman, to whom are due the Society's heartfelt thanks. Under his inspiring leadership the meetings were fairly well attended, and quite a good deal was accomplished.

All of us are, of course, busy at our work, and some of us put in rather long hours. We ought, however, to think enough of our Society to put in a little work on its affairs now and then. There are some that do not attend even one meeting of the committee on which they have been appointed.

That is the wrong attitude to take. The Boston Society of Civil Engineers will thrive exactly in direct proportion to the interest that its members take in its activities. Every member must put some energy into the affairs of the Society, even if it amounts only to criticizing the officers, — who, by the way, do not deserve any, handicapped as they are, — or the Society will become merely a collection of fossils.

Not one of us considers ourself a fossil, — that is sure, — yet in our actions we proclaim ourselves so.

II

While we are on the subject of committees, let us consider some possible improvements. Three suggestions at once spring to mind.

Every committee should have a chairman and a vice-chairman, so that if the chairman is out of town or cannot attend a meeting the committee will function just the same, for the vice-chairman can act as chairman *pro tempore.*

Every committee should be required to hold at least four meetings a year. The get-together alone will be worth the effort expended in attending the meetings, and once the committee is together, maybe some work will be done.

We ought to break up the Committee on Membership and Publicity into two, one on membership and the other on public relations; or the publicity part of it should be turned over to the Committee on Publications.

III

All the above is not said in a spirit of criticism of any person or committee. The intention is merely to be helpful. The officers of the Society are overburdened with work, while the committees (with the brilliant exception of last year's Library Committee) who should help carry the burden seem to do very little to help. The "high jobs" are not all honor — they are a lot of hard work. A member's duty does not end with the payment of his dues.

All of us are trying to raise the engineering profession in the public's esteem. It is our great pride that there are very few cases of engineers going wrong — far fewer, apparently, than clergymen. The public is just beginning to take an interest in the engineer. This interest is kindly, and to deserve it our interest in our Society must also be kindly. We must pull together, and if necessary make some sacrifice for the cause, as it were. The sacrifice required of us is to help our officers by attending committee meetings and doing some work. Surely, that is not too hard for us.

Low Water Conditions in the New England District

By H. B. Kinnison *

Beginning with the first of June, 1929, the New England district has experienced one of the most severe droughts in recent years. The previous year was one of ample precipitation, and the river stages in the spring of 1929 were of average height. Soon after the beginning of the dry period, however, the rivers dropped below their normal levels, and by early summer they had reached low water stage. An exceedingly dry summer and fall caused the river stages to drop even more, so that before the usual fall increase in stage, due to the secession of growth of vegetation, the streams had reached extremely low stages. The drought lasted throughout the winter, very little snow falling in New England, except in the northernmost parts of Vermont and New Hampshire, where they had an average snowfall. Records of flow of rivers in this northern section indicate that it has not been affected by the drought; in fact, the precipitation at Burlington, Vermont, is somewhat greater than normal for the year ending June 30, 1930. The Merrimack and Connecticut Rivers, which rise in this northern section, have not shown the tendency to drop below average low water stages, due to the more ample rainfall.

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* District Engineer, Water Resources Branch, United States Geological Survey, Custom House, Boston.

Table 1 shows the yearly yield of various watersheds in second-feet per square mile, and the relation of this unit to the normal yield. This table shows that the yield of the Wachusett Watershed is but 44 per cent of the average of a thirty-three year period, and during this period there has never been a year when the yield was so low. Also it will be seen from Table 2 that the rainfall for the year ending June 30, 1930, was but 65 per cent of the yearly average. As a result of the extremely low yield of the Wachusett Watershed, the Wachusett Reservoir, which supplies water to metropolitan Boston, is rapidly being depleted, and unless relief is afforded in the near future the reservoir will drop to the lowest level for this period of the year since it was completed, in 1908.

The water supply for many communities in southern New Hampshire, and throughout Massachusetts, is becoming dangerously low. The State boards of health are having considerable difficulty in preventing contamination of city water supplies by the use of contaminated lake water.

Table 2 gives the yearly rainfall for several of the lowest years of record at a number of stations, and shows how these compare with the normal rainfall. This table indicates that the precipitation for the year ending June 30, 1930, at Lowell, Massachusetts, was the lowest since 1836, when a yearly total of 24.9 inches was recorded, which was the minimum since 1826, when the record was started. The record of rainfall data on the Wachusett and Sudbury Watersheds is the average for four stations, and is the lowest yearly average since the records were started, which on the Sudbury was fifty-five years ago.

TABLE 1. — RIVER DISCHARGE IN SECOND-FEET PER SQUARE MILE FOR YEARLY PERIODS DURING SEASONS OF LOW FLOW

STATION	Year ending —	Yield in Second-Feet per Square Mile	Per Cent of Average
Wachusett River, at Clinton, Mass. (33-year period)	June 30, 1930	.74	44
	June 30, 1911	.79	47
	Dec. 31, 1911	1.055	63
	Nov. 30, 1917	1.24	73
Mean for 33 years	. . . .	1.689	100
Cochituate River, near Cochituate, Mass. (67-year record)	June 30, 1930	.65	45
	Oct. 30, 1871	1.03	72
	Jan. 31, 1881	.71	49
	Nov. 30, 1883	.72	50
	Jan. 31, 1893	.93	65
	Feb. 28, 1895	.91	63
	Mar. 31, 1911	.64	44
	July 31, 1911	.59	41
	Dec. 31, 1880	.727	50
	Dec. 31, 1883	.719	50
	Dec. 31, 1894	.951	66
	Dec. 31, 1908	.964	67
	Dec. 31, 1911	.871	60
Mean for 67 years	. . .	1.44	100
Sudbury Watershed, Mass.	June 30, 1930	.66	–
	Dec. 31, 1880	.895	–
	Dec. 31, 1883	.824	–
Merrimack River, at Lawrence, Mass. (51-year record)	June 30, 1930	1.00	–
	Sept. 30, 1883	.942	–
	Sept. 30, 1895	1.00	–
	Sept. 30, 1911	.786	–
Housatonic River, near Great Barrington, Mass. (15-year record)	June 30, 1930	1.20	–
	Sept. 30, 1915	1.36	–
Connecticut River, at Orford, N. H. (29-year record)	June 30, 1930	1.42	–
	Sept. 30, 1911	1.29	–
	Sept. 30, 1915	1.34	–
Souhegan River, at Merrimack, N. H. (19-year record)	June 30, 1930	.875	–
	Sept. 30, 1911	.839	–
Ware River, at Gibbs Crossing, Mass. (18-year record)	June 30, 1930	.756	–
	Sept. 30, 1925	1.02	–

TABLE 2. — RAINFALL FOR PERIODS DURING SEASONS OF DROUGHT

LOCATION	Year ending —	Rainfall in Inches	Per Cent of Average
Average for Massachusetts	May 31, 1930	32.87	74
Period of several years		44.45	–
Lowell, Mass.	June 30, 1930	26.62	60
	May 31, 1915	27.219	61
	Sept. 30, 1883	28.123	63
	June 30, 1911	28.29	64
	May 31, 1836	24.90	56
	Dec. 31, 1846	28.03	63
Wachusett Watershed (33-year record)	June 30, 1930	29.48	65
	May 31, 1925	35.51	79
Average for 33 years		45.15	100
Sudbury Watershed	June 30, 1930	30.78	69
	May 31, 1925	37.19	84
	Dec. 31, 1909	37.91	85
	Dec. 31, 1910	35.90	81
	Dec. 31, 1911	35.46	80
Average for 55 years		44.48	100
Concord, N. H.	June 30, 1930	25.79	67
	Dec. 30, 1894	27.64	71
	Dec. 30, 1899	28.57	74
	Dec. 30, 1908	26.25	68
	Dec. 30, 1913	29.04	75
	Dec. 30, 1924	28.65	74
Average for 72 years		38.80	100
Burlington, Vt.	June 30, 1930	32.73	101
	Dec. 31, 1854	25.45	78
	Dec. 31, 1873	25.92	80
	Dec. 31, 1879	24.27	75
	Dec. 31, 1880	25.26	78
	Dec. 31, 1882	25.64	79
	Dec. 31, 1894	22.96	71
	Dec. 31, 1908	23.49	72
	Dec. 31, 1913	25.75	79
	Dec. 31, 1914	22.62	70
	Dec. 31, 1915	25.68	79
Average for 94 years		32.55	100

Table 2. — Rainfall for Periods during Seasons of Drought — *Concluded*

Location	Year ending —	Rainfall in Inches	Per Cent of Average
St. Johnsbury, Vt.	June 30, 1930	30.84	88
	Dec. 31, 1894	27.15	77
	Dec. 31, 1904	29.67	85
	Dec. 31, 1909	29.78	85
	Dec. 31, 1921	27.32	78
Average for 35 years	. . .	34.86	100
Boston, Mass. (Cambridge)	June 30, 1930	29.89	66
	Dec. 31, 1762	24.47	54
	Dec. 31, 1822	27.20	60
	Dec. 31, 1828	32.41	71
	Dec. 31, 1837	33.52	74
	Dec. 31, 1846	29.95	66
	Dec. 31, 1887	33.75	74
	Dec. 31, 1899	34.69	76
	Dec. 31, 1902	33.93	75
	Dec. 31, 1905	32.08	71
	Dec. 31, 1908	30.07	66
	Dec. 31, 1910	28.33	62
	Dec. 31, 1912	34.55	76
	Dec. 31, 1914	34.31	73
	Dec. 31, 1918	34.40	76
	Dec. 31, 1923	34.91	77
	Dec. 31, 1924	34.21	75
Average for 96 years	. . .	45.46	100
Springfield, Mass.	Apr. 30, 1930	27.35	63
	Dec. 31, 1864	30.90	71
	Dec. 31, 1905	31.02	71
	Dec. 31, 1912	31.17	71
	Dec. 31, 1924	31.12	71
Average for 81 years	. . .	43.76	100
Hartford, Conn.	Sept. 30, 1930	36.65	85
	Dec. 31, 1914	34.30	79
	Dec. 31, 1921	34.50	80
	Dec. 31, 1924	33.54	77
Average for 30 years	. . .	43.30	100

X. Henry Goodnough, Chief Engineer of Massachusetts Department of Health, retires after Forty-five Years of Public Service

Dinner Given in his Honor at Engineers' Club

In recognition of the long and distinguished career of X. Henry Goodnough, who retired on October 23, 1930, from the position of Director and Chief Engineer of the Sanitary Engineering Division of the Massachusetts Department of Public Health after forty-five years of service, thirty years of which were as Chief Engineer, a dinner was held in his honor at the Engineers' Club, Boston, on Thursday evening, November 13, 1930.

This dinner was attended by one hundred friends and admirers of his service, including heads of State departments, civil and sanitary engineers, chemists, members of the medical and legal professions, and city and town officials, both of this and other states.

Mr. Frank E. Winsor, Chief Engineer of the Metropolitan District Water Supply Commission, was toastmaster. Dr. George H. Bigelow, Commissioner of Public Health, extended the greetings of His Excellency, Frank G. Allen, Governor of the Commonwealth, and expressed appreciation of the part taken by Mr. Goodnough in the development of public health control in this State. Other speakers included Mr. J. Waldo Smith, Consulting Engineer and former Chief Engineer of the Board of Water Supply of New York City; Mr. Charles P. Howard, chairman of the Massachusetts Commission on Administration and Finance; Mr. Warren J. Scott, Director of Sanitary Engineering of the Connecticut Department of Public Health; Mr. Davis B. Keniston, Commissioner of the Metropolitan District Commission; Mr. Harry W. Clark, Director and Chief Chemist of the Division of Water and Sewage Laboratories of the Massachusetts Department of Public Health; Mr. James L. Tighe, Consulting Engineer, Holyoke, and member of the Massachusetts Public Health Council.

The appreciation of the services of Mr. Goodnough, and the regard of all for him are well summarized in the following remarks by Mr. James L. Tighe:

There have been so many nice things said about our guest of the evening, all so well deserved, that I have been wondering if anything has been left for me to say.

We are meeting here in a friendly, informal and cheerful fashion, happy to be here to signify our goodwill toward our guest of honor. It is a very pleasant occasion, yet it seems to me that beneath our appearances of enjoyment there is another tone, another feeling, which in reality predominates this gathering. We are pleased because we are expressing by our presence our esteem and our admiration for our friend Mr. Goodnough, and because our attendance is heartfelt and not forced in any sense.

Before speaking of the relation of our honored guest to the municipal engineer and engineer in private practice, I take this opportunity to express my own satisfaction in being here to honor one who is especially deserving of honor.

This meeting is a tribute to him for his professional attainments, for his achievements in the great work which has occupied practically all of his working life, and, above all, for the splendid attributes which make up his character.

No one knows these qualities better than the municipal engineer and engineer in private practice. In all their work pertaining to sanitary engineering he has been to them the guide, the adviser and the friend for nearly half a century, or since the days when he was one of the pioneers in the advancement of public health, for the creation of better water supply and sewerage systems. When they were confronted with difficult sanitary problems Mr. Goodnough's counsel always led them to safety and success.

It is nearly forty years since my first meeting with Mr. Goodnough. Since that time I have had frequent contact with him, and during the last six years, while attending to the duties of the Public Health Council of the State, our associations have been more frequent. It is through associations extending over a period of years that a man learns to know the real worth of a fellow man.

There is no need to tell tonight of the great work he has accomplished in his public office, and the great benefit this work has been to the people of the State. You are all well familiar with it. What city or town throughout the Commonwealth has not been the beneficiary of his knowledge and of his wisdom? Outside the Commonwealth, too, has not his leadership always been recognized? His record and standing in his profession reflect his abilities. To know these things simply show to us the man himself whom we are so proud to honor tonight.

In his long years of service to the State, under his supervision, he has seen a great development in sanitary engineering. In these years there was for him an enormous amount of study, of reflection and advice involved. Always judicial, what was good he fought for and won; what was bad he opposed and prevented. He had battles to fight, too, and I believe he has the unique distinction of never losing one.

Mr. Goodnough always had a prodigious memory, and, because of this, there is not a community in the State, even the smallest, for which he cannot tell offhand its sanitary status. Neither is there a public or private water supply or sewerage system that he is not able to describe from memory. He could even describe the roads and lanes that lead to them, although he may not have been over these roads for twenty or thirty years. Our honored guest needed to go over the road but once, and thereafter it was indelible in his mind, never to be forgotten.

The right road to follow in life, which has led him to success and distinction, was also indelibly defined in his mind.

Neither his calm demeanor, his scholarly attainments, his unassuming manner or anything else can disguise to his friends the man they know and value with a sincerity which comes from a full understanding of his worth.

Whether he approves of it or not the municipal engineer and the engineer in private practice are here to say they admire Mr. Goodnough as a man, honor him as an eminent engineer, are grateful for his assistance, advice and leadership — and as a friend, and simply, that they like him.

Mr. Goodnough acknowledged appreciatively the honor bestowed upon him, and in retrospect reviewed some of the interesting developments in public health matters, and particularly his associations with men of prominence in this field of endeavor, and stated that much of his success was due to those he was able to have as assistants in his work.

At the close of the dinner those present individually extended Mr. Goodnough their best wishes and satisfaction that his plans for continuing in practice as a consulting engineer would make his experience available to the profession.

The dinner was arranged by a committee consisting of Charles W. Sherman, Chairman, George H. Bigelow, Frank E. Winsor, Richard K. Hale, Lewis E. Moore, Frank B. Walker and George H. Finneran.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Designers Section

NOVEMBER 12, 1930. — A regular meeting of the Designers Section was called to order at 6.15 P.M. by the Chairman, Norman P. Randlett, in the rooms of the Engineering Societies of Boston.

The speaker for the evening was Mr. C. K. Stiff, Engineer of the Associated Factory Mutual Fire Insurance Companies. Mr. Stiff discussed the fire protection of structural members. Numerous conditions were considered, and the different cases were illustrated by lantern slides showing effects of heat on tested specimens. There were also slides of diagrams and tables prepared by the Insurance Companies' organization.

Many of the eighteen members and guests present took part in the discussion which followed the paper. The meeting adjourned at 8 P.M.

LAWRENCE G. ROPES, *Clerk.*

APPLICATIONS FOR MEMBERSHIP

[Dec. 20, 1930]

THE By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission

ANDERSON, ALFRED CHARLES, East Foxborough, Mass. (Age 20, b. Boston, Mass.) A student at Northeastern University, and during co-operative periods has worked for Reed & Son, Benton, N. H.; for Fraser-Brace Contracting Company, on the Connecticut River Development, and as chief of survey party on land court and road surveys, with Herbert Richardson. Refers to *H. B. Alvord, C. O. Baird, J. W. Ingalls, C. A. White.*

AUBEY, MILLARD HENRY, Framingham, Mass. (Age 20, b. Framingham, Mass.) In May, 1927, was employed as axman by the Metropolitan District Water Supply Commission, and in December of that year his rating was changed to that of rodman, when he was transferred to field party in charge of the Coldbrook-Wachusett Tunnel. He worked here until he entered Northeastern University in September, 1928. During his co-operative periods in school he returned to this same position as a rodman and transitman. Refers to *H. B. Alvord, C. O. Baird, J. W. Ingalls, W. E. Nightingale.*

BAGLOE, WILLIAM ALEXANDER, Brighton, Mass. (Age 20, b. Brighton, Mass.) Entered Mechanic Arts High School in

September, 1923, and graduated in June, 1927; in February, 1928, entered Northeastern University. During co-operative periods has worked for the city of Newton as transitman. Refers to *H. B. Alvord, C. O. Baird, W. H. Fitzgerald, J. H. Lowe, G. A. A. Morse.*

Bateman, Charles Edward, North Stratford, N. H. (Age 22, b. Glenn, N. H.) Graduated from Stratford High School in 1925. Worked for Oakland Motor Car Company, in Pontiac, Mich. for ten months, finishing in December, 1926. In United States Army until April, 1928, and worked for A. H. Parker Company from April, 1928, to December, same year. Entered Northeastern University in January, 1929. Refers to *W. J. Alcott, H. B. Alvord, C. O. Baird, J. W. Ingalls.*

Brown, Roger Shaw, Lynn, Mass. (Age 21, b. Lynn, Mass.) Graduate of Lynn English High School in 1928, and entered Northeastern University in January, 1929. During the co-operative periods has worked for Aspinwall & Lincoln as rodman, for the city engineer of Lynn, where he has been employed since December, 1929. Refers to *W. J. Alcott, H. B. Alvord, C. O. Baird, J. W. Ingalls.*

Burton, Jr., William H., Somerset, Mass. (Age 20, b. Somerville, Mass.) Graduate of Somerset High School and entered Northeastern University with the class of 1932. During co-operative periods worked for Fall River Electric Light, meter department. Refers to *W. J. Alcott, H. B. Alvord, C. O. Baird, J. W. Ingalls.*

Carlson, John George, Milford, Mass. (Age 21, b. Milford, Mass.) He is a senior at Northeastern University, and during co-operative periods has been employed by the town of Framingham as a rodman one month; as a transitman by Wilcot & Sons, one month; steel rodman four months for Fraser-Brace Contracting Company, and in charge of steel fabricating for six weeks. Refers to *H. B. Alvord, C. O. Baird, J. W. Ingalls, C. A. White.*

Coffey, Joseph Henry, Bradford, Mass. (Age 21, b. Haverhill, Mass.) Entered Northeastern University in September, 1927, and during co-operative periods has been employed by Rickard Shoe Company, Cobb, Beesley & Miles, engineers in Springfield, and the Public Works Department, city of Melrose. Refers to *H. B. Alvord, C. O. Baird, J. W. Ingalls, H. O. Scott.*

Condon, Eugene Bernard, Roslindale, Mass. (Age 23, b. Roxbury, Mass.) Educated at Boston English High School and now a student at Northeastern University. Refers to *H. B. Alvord, C. O. Baird, J. W. Ingalls, J. H. Lowe.*

Forsberg, Algot Oscar, Worcester, Mass. (Age 24, b. Worcester, Mass.) Entered Northeastern University in January, 1928, and during co-operative periods has been employed by Northeastern University in physics department as laboratory assistant for two years; city of Worcester as rodman three summers; New Haven Railroad as chainman; at present, draftsman in Department of Public Health on special commission on harbor investigation. Refers to *H. B. Alvord, C. O. Baird, J. W. Ingalls, C. A. White.*

Gilson, Kenneth Holden, Stoneham, Mass. (Age 21, b. Stoneham, Mass.) Graduate of Stoneham High School in June, 1927, and now a student of Northeastern University. Refers to *H. B. Alvord, C. O. Baird, J. W. Ingalls, W. E. Nightingale.*

Koff, David Edward, Lynn, Mass. (Age 23, b. Chelsea, Mass.) Is a student at Northeastern University, and during co-operative periods has a position with the Massachusetts Department of Public Works. Refers to *H. B. Alvord, C. O. Baird, E. R. Goodwin, J. H. Lowe.*

Ledgard, Francis H., Maynard, Mass. (Age 20, b. Maynard, Mass.) Graduate of Maynard High School in June, 1928, and entered Northeastern University in January, 1929. During the summer of 1930 worked for E. N. Montague, C.E. Refers to *H. B. Alvord, C. O. Baird, J. H. Lowe, E. N. Montague, B. B. Sheridan.*

Norton, George Kenneth, Boston, Mass. (Age 23, b. Pawlet, Vt.) Graduate of Granville High School, and at present a student at Northeastern University. During co-operative periods has been employed by the city of Medford, city of Waltham, towns of Milton and Walpole. Refers to *H. B. Alvord, C. O. Baird, J. W. Ingalls, W. E. Nightingale.*

Parsons, Jr., Edward J., Gloucester, Mass. (Age 22, b. Gloucester, Mass.) Is a student at Northeastern University, and during co-operative periods has worked with the city engineer, Gloucester; Connecticut River Development Company, on the Fifteen Miles Falls Development in Vermont and New Hampshire. Refers to *W. J. Alcott, H. B. Alvord, C. O. Baird, J. W. Ingalls.*

Patten, Robert Vernon, Hyde Park, Mass. (Age 20, b. Hyde Park, Mass.) Graduate of Hyde Park High School in 1927, and entered Northeastern University in September, 1927. During co-operative periods has been in the Engineering Department of the city of Waltham. Refers to *H. B. Alvord, C. O. Baird, J. W. Ingalls, C. A. White.*

Perry, Robert Walker, Cohasset, Mass. (Age 21, b. Cohasset, Mass.) Graduated from Cohasset High School and entered Northeastern University in 1928. During co-operative periods was employed by E. W. Branch of Quincy; L. W. Perkins of North Scituate, to date. Refers to *H. B. Alvord, C. O. Baird, J. W. Ingalls, C. A. White.*

Scher, Charles Victor, Beverly, Mass. (Age 21, b. Beverly, Mass.) Has completed three years at Northeastern University and now in his senior year. During the co-operative periods has been employed by Aspinwall & Lincoln as a rodman for one year and as transitman for the balance of the time. Refers to *H. B. Alvord, C. O. Baird, W. H. Connor, J. W. Ingalls, C. A. Wolfrum.*

Smith, Merton C., Amherst, Mass. (Age 22, b. Amherst, Mass.) Graduated from Amherst High School in 1926, completed a postgraduate course in Greenfield High School, and then entered Northeastern University. For fourteen months previous to and during postgraduate work in Greenfield was employed by F. Dean Avery, C.E. During the co-operative periods at the University and during the summers has been employed by Cobb Beesley & Miles, engineers, of Springfield. Refers to *H. B. Alvord, C. O. Baird, J. W. Ingalls, C. A. White.*

Thompson, Theron Barker, Stoneham, Mass. (Age 21, b. Somerville, Mass.) Graduate of Stoneham High School, and now a student at Northeastern University. During co-operative periods has been employed by John H. Rand, C.E., Melrose, from November, 1929, to May, 1930; as senior engineering aide in the Massachusetts Department of Public Works from September, 1930, to date. Refers to *H. B. Alvord, C. O. Baird, J. W. Ingalls, W. E. Nightingale.*

Whitaker, Albert P., Boston, Mass. (Age 18, b. Leicester, Mass.) Entered Northeastern University in January, 1929. During co-operative periods worked for Fraser-Brace Company on Fifteen Miles Falls lower development at East Barnet, Vt., working for the period of ten weeks until August, 1930; from September, 1930, worked for G. A. Griffin, C.E., at Woods Hole, Mass., drafting and some surveying. Refers to *H. B. Alvord, C. O. Baird, J. W. Ingalls, C. A. White.*

DEATH

Edward Warren Henck, died October 26, 1930.

INDEX

VOLUME XVII, 1930

Abbreviation: I. — Illustrated

Names of authors are printed in *italics*

PAGE

Akers, Kenneth F. Fire Protection Engineering as Applied to Municipalities . . . I., April, 109
Allen, C. Frank. Discussion of Recent Terminal Improvements of the Boston & Maine Railroad . . . Jan., 29
American Society of Civil Engineers' Prize Awards . . . Nov., 538
Angular Rotation Method for the Analysis of Multiple Span Frames. *R. A. Freeman* and *A. S. Gelman* . . . I., Dec., 543
Annual Reports of the Society . . . April, 146
Awarded to the Lowest Bidder. *Edward Grossman* . . . May, 249

Babcock, John B., 3d. Discussion of Recent Terminal Improvements of the Boston & Maine Railroad . . . Jan., 29
Babcock, John B., 3d. M. I. T. Offers Course in Railroad Operation . Jan., 32
Barrows, H. K. Discussion of Stream-Flow Data — Its Collection and Use . . . May, 242
Barrows, H. K. Review of "Flood Flows," by *Allen Hazen* . . May, 255
Barrows, H. K. Discussion of Hydraulic and Mechanical Features of the Bellows Falls Hydro-Electric Plant of the New England Power System . . . March, 92
Bell, W. R. Discussion of Hydraulic and Mechanical Features of the Bellows Falls Hydro-Electric Plant of the New England Power System . . . March, 96
Bellows Falls Hydro-Electric Plant of the New England Power System, Hydraulic and Mechanical Features. *Edwin A. Dow* . I., March, 71
Bidder, Award to the Lowest. *Edward Grossman* . . . May, 249
Bigelow, W. W. Discussion of Design of Wind Bracing . . . Nov., 525
Booker, C. A. and *H. O. Weber.* The Design of Supporting Structures for Transmission Lines . . . I., Feb., 37
Boston, Some Changes since it was Settled Three Hundred Years Ago. President's Address. *Frank E. Winsor* . . . April, 103
Boston & Maine Railroad, Recent Terminal Improvements. *Frank C. Shepherd* . . . I., Jan., 1
Bradford, Frank. Fire Protection Engineering as Applied to Municipalities . . . April, 131

PAGE

Brandt, Robert. Discussion of Hydraulic and Mechanical Features of Bellows Falls Hydro-Electric Plant of the New England Power System . . . . March, 96
Bryant, William A. Discussion of Fire Protection Engineering as Applied to Municipalities . . . . April, 133
Bugbee, Percy. Discussion of Fire Protection Engineering as Applied to Municipalities . . . . April, 128
Building Code, Uniform, Suggested for New England . . . April, 137

Cameron, Edward H. A Better Journal . . . . . . April, 135
Charlestown Water Works (Mystic System), 1865. *Charles W. Sherman* . . . . June, 280
Clair, Miles N. Ready-Mixed Concrete . . . . . . Oct., 474
Committees that Do not "Committ." By a Committeeman . . Dec.,
Concrete, Ready-Mixed. *Miles N. Clair* . . . . . . Oct., 474
Concrete, Exposed Surfaces and their Protection. *Angus B. MacMillan* . . . . Oct., 467
Crandall, J. Stuart. "Le Port de New York dans son Rôle Économique." *Gaston Eyskens.* A review . . . . May, 254

Design of Supporting Structures for Transmission Lines. *C. A. Booker* and *H. O. Weber* . . . . I., Feb., 37
Dow, Edwin A. Hydraulic and Mechanical Features of Bellows Falls Hydro-Electric Plant of the New England Power System . I., March, 71

Examples of Design for Wind Bracing in Tall Buildings. *S. C. Jemian* . . . . I., Dec., 563
Exposed Surfaces of Concrete and their Protection. *Angus B. MacMillan* . . . . Oct., 467

Fay, Frederic H. The Port of Hamburg . . . . . . I., Oct., 481
Fay, Spofford & Thorndike. Awarded prize for Design of Lake Champlain Bridge . . . . Sept., 465
Fire Protection Engineering as Applied to Municipalities. *Kenneth F. Akers* . . . . I., April, 109
Fiske, Henry A. Discussion of Fire Protection Engineering as Applied to Municipalities . . . . April, 130
"Flood Flows." *Allen Hazen*, reviewed by *H. K. Barrows* . . May, 255
Floods, Report of Committee on . . . . I., Sept., 285
Freeman, R. A., and *A. S. Gelman.* The Angular Rotation Method for the Analysis of Multiple Span Frames . . . . . I., Dec., 543

Gelman, A. S., and *R. A. Freeman.* The Angular Rotation Method for the Analysis of Multiple Span Frames . . . . . I., Dec., 543
Goodnough, X. Henry. Dinner in Honor of . . . . . Dec., 572
Graustein, Edward A. Discussion of Stream-Flow Data — Its Collection and Use . . . . May, 245
Grossman, Edward. Awarded to the Lowest Bidder . . . May, 249
Grossman, Edward. Vitruvius . . . . Feb., 66

PAGE
Hale, Richard Augustus. Memoir . . . . April, 139
Hamburg, Port of. *Frederic H. Fay* . . . . . . I., Oct., 481
Hersum, L. R. Discussion of Design of Wind Bracing . . . Nov., 531
How Shall the Water Front be Developed? *Roy S. MacElwee* . March, 98
Hultman, Eugene C. Discussion of Fire Protection Engineering as Applied to Municipalities . . . . April, 126
Hydraulic and Mechanical Features of Bellows Falls Hydro-Electric Plant of the New England Power System. *Edwin A. Dow* . I., March, 71

Japan. Engineering and the Industries of . . . . . I., June, 257
Jemian, S. C. Discussion of Design of Wind Bracing . . . Nov., 531
Jemian, S. C. Examples of Design for Wind Bracing in Tall Buildings I., Dec., 563
Johnson, Lewis J. Discussion of Design of Wind Bracing . . Nov., 534
Journal, A Better. *Edward H. Cameron* . . . . . I., April, 135

Kinnison, H. B. Low Water Conditions in the New England District . . . . Dec., 567
Kinnison, H. B. Stream-Flow Data — Its Collection and Use . May, 171

Lacount, H. O. Fire Protection Engineering as Applied to Municipalities . . . . April, 132
Lake Champlain Bridge, Boston Engineers Awarded Prizes for Design Sept., 465
"Le Port de New York dans son Rôle Économique." *Gaston Eyskens*, reviewed by *Stuart Crandall* . . . . . . May, 254
Low Water Conditions in the New England District. *H. B. Kinnison* Dec., 567

Main, Charles T. Engineering and the Industries of Japan . . I., June, 257
M. I. T. Offers Course in Railroad Operation. *John B. Babcock, 3d* . Jan., 32
McClintock, Miller. Fire Protection Engineering as Applied to Municipalities . . . . April, 125
MacElwee, Roy S. How Shall the Water Front be Developed? . March, 98
MacMillan, Angus B. Exposed Surfaces of Concrete and their Protection . . . . Oct., 467
Martin, Sidney A. Discussion of the Design of Supporting Structures for Transmission Lines . . . . I., Feb., 60
Moore, Lewis E. Discussion of the Design of Wind Bracing . . Nov., 524
Moore, Lewis E. Discussion of Recent Terminal Improvements of the Boston & Maine Railroad . . . . . . Jan., 29
Multiple Span Frames, the Angular Rotation Method for Analysis. *R. A. Freeman* and *A. S. Gelman* . . . . . I., Dec., 543

Osgood, I. Discussion of Fire Protection Engineering as Applied to Municipalities . . . . April, 130

Pierce, John F. Discussion of Fire Protection Engineering as Applied to Municipalities . . . . April, 134
Pike, Waldo F. Discussion of Design of Wind Bracing . . . . Nov., 524

PAGE

President's Address. Some Changes since Boston was Settled Three Hundred Years Ago. *Frank E. Winsor* . . . . April, 103
Proceedings of the Society:
Jan., 34; Feb., 68; March, 101; April, 140; May, 252; June, 281; Sept., 465; Oct., 490; Nov., 539; Dec., 574.

Rablin, John Richard, Memoir . . . . April, 137
Railroad Operation, M. I. T. Offers a Course. *John B. Babcock, 3d* . Jan., 32
Ready-Mixed Concrete. *Miles N. Clair* . . . . . Oct., 474
Recent Terminal Improvements of the Boston & Maine Railroad. *Frank C. Shepherd* . . . . I., Jan., 1

Safford, Arthur T. Chairman, Committee Reporting on Floods . I., Sept., 285
Safford, Truman, H. Discussion of Stream-Flow Data — Its Collection and Use . . . . May, 247
"Sewerage and Sewage Disposal," reviewed. *Robert Spurr Weston* . April, 144
Shepherd, Frank C. Recent Terminal Improvements of the Boston & Maine Railroad . . . . I., Jan., 1
Sherman, Charles W. The Charlestown Water Works (Mystic System), 1865 . . . . June, 280
Smulski, Edward. Design of Wind Bracing . . . . . I., Nov., 491
Snow, J. Parker. Discussion of Recent Terminal Improvements of the Boston & Maine Railroad . . . . Jan., 30
Some Changes since Boston was Settled Three Hundred Years Ago. President's Address. *Frank E. Winsor* . . . . April, 103
Stream-Flow Data — Its Collection and Use. *H. B. Kinnison* . I., May, 171

Transmission Lines, the Design of Supporting Structures. *C. A. Booker* and *H. O. Weber* . . . . I., Feb., 37
Turner, Howard M. Discussion of Stream-Flow Data — Its Collection and Use . . . . May, 243

Uniform Building Code Suggested for New England . . . April, 137

Varney, Edward A. Discussion of Design of Wind Bracing . . Nov., 526
Vitruvius. *Edward Grossman* . . . . Feb., 66

Water Front, How Shall it be Developed? *Roy S. MacElwee* . March, 98
Weaver, Frederic N. Discussion of Design of Wind Bracing . . Nov., 530
Weber, H. O., and *C. A. Booker.* The Design of Supporting Structures for Transmission Lines . . . . I., Feb., 37
Weston, Robert Spurr. Review of "Sewerage and Sewage Disposal," by *Leonard Metcalf* and *Harrison P. Eddy* . . . . . April, 144
Wind Bracing, Design of. *Edward Smulski* . . . . . I., Nov., 491
Wind Bracing in Tall Buildings, Examples of Design. *S. C. Jemian* Dec., 563
Winsor, Frank E. Some Changes since Boston was Settled Three Hundred Years Ago. Address of President . . . April, 103

Vol. XVII DECEMBER, 1930 No. 10

BOSTON SOCIETY OF CIVIL ENGINEERS

FOUNDED 1848

INDEX TO ADVERTISERS

PAGE

Algonquin Engraving Co., Inc., 18 Kingston St., Boston . . . v
Aspinwall & Lincoln, 46 Cornhill, Boston . . . iii
Barbour, Frank A., Tremont Building, Boston . . . iv
Barrows, H. K., 6 Beacon St., Boston . . . iii
Bay State Dredging and Contracting Co., 62 Condor St., East Boston . . . ix
Bigelow, Kent, Willard & Co., Park Square Bldg., Boston . . . iv
Bishop, J. W., Co., 100 Erie St., Cambridge . . . x
Black & Blue Print Co., 71a Pearl St., Boston . . . xii
Blakeslee, Rollins Corp'n, 6 Beacon St., Boston . . . v
Bruno & Petitti, Contractors, 18 Tremont St., Boston . . . viii
Bryant, Henry F., & Son, 334 Washington St., Brookline . . . v
Buff & Buff Co., Jamaica Plain . . . vi
Carleton-Mace Engineering Corp., 252 Dover St., Boston . . . xiii
Chase, Maurice E., 263 Summer St., Boston . . . viii
Coleman Bros., 245 State St., Boston . . . vi
Collins, J. Lincoln, 120 High St., Boston . . . xiii
Comstock & Wescott, 80 Federal St., Boston . . . vii
Crandall Engineering Co., 134 Main St., Cambridge . . . xii
Crosby, Irving B., 6 Beacon St., Boston . . . iii
Donoghue Co., P. W., 170 Purchase St., Boston . . . v
Ellis, W. H., & Son Co., East Boston . . . viii
Fairchild Aerial Surveys, 260 Tremont St., Boston . . . ii
Fay, Spofford & Thorndike, 44 School St., Boston . . . iii
Foundation & Construction Co., 108 Massachusetts Ave., Boston . . . vii
Fuller & McClintock, 170 Broadway, New York . . . iii
Gibson & Williams, Inc., 65 Long Wharf, Boston . . . vi
Goss, John L., Corporation, 77 Summer St., Boston . . . x
Gow Company Inc., 956 Park Square Building, Boston . . . ix
Grande, V., 100 Academy Hill Road, Boston . . . vi
Granite Paving Block Mfrs.' Assoc., 31 State St., Boston . . . xi
Graustein, Edward A., 6 Beacon St., Boston . . . iv
Holzer, U., Inc., 333 Washington St., Boston . . . vii
Hughes, Edward F., 53 State St., Boston . . . vii
Jackson & Moreland, Park Square Building, Boston . . . iii
Kennedy, John F., & Co., 80 Federal St., Boston . . . xi
Lanza, Frank, & Sons, 608 Pemberton Bldg., Boston . . . vii
Leighton-Mitchell Co., 99 Chauncy St., Boston . . . xiii
Ley, Fred T., & Co., Boston . . . xiii
Lunt Moss Corp., 43 South Market St., Boston . . . v
Main, Inc., Chas. T., 201 Devonshire St., Boston . . . iv
Makepeace, Inc., B. L., 387 Washington St., Boston . . . Back cover
Marden & Orlando Co., 46 Cornhill, Boston . . . vii
McGann, T. F., & Sons Co., 120 Portland St., Boston . . . vii
Metcalf & Eddy, Statler Building, Boston . . . iii
Monahan Pump Equipment Co., 10 High St., Boston . . . vi
Monks & Johnson, 99 Chauncy St., Boston . . . iv
Montan Treating Co., 141 Milk Street, Boston . . . vi
Moore, Lewis E., 73 Tremont St., Boston . . . v
Mulcare, Thomas, Inc., 66 Western Ave., Boston . . . v
New England Concrete Pipe Corp., 79 Milk St., Boston . . . viii
New England Foundation Co., Inc., 38 Chauncy St., Boston . . . vi
New England Power Association, 89 Milk St., Boston . . . x
New England Wood Preserving Co., Chamber of Commerce Bldg., Boston . . . x
Old Corner Book Store, 50 Bromfield St., Boston . . . vi
Packard, George A., 50 Congress St., Boston . . . iv
Peters, A. C., Co., 46 Cornhill, Boston . . . xiii
Pike, Edward N., 38 Chauncy St., Boston . . . iv
Reed, Alonzo B., 88 Broad St., Boston . . . v
Reidy, Maurice A., 44 School St., Boston . . . xiii
Rendle, Roy B., & Co., 364 Border St., East Boston . . . vi
Rideout, Chandler & Joyce, 130 Broad St., Boston . . . vi
Scully Co., 238 Main St., Cambridge . . . vii
Shaw, Frank R., 161 Devonshire St., Boston . . . viii
Simpson Bros. Corpn., 77 Summer St., Boston . . . vii
Southern Surety Co. of New York, 89 Broad St., Boston . . . v
Stuart, T., & Son Co., 70 Phillips St., Watertown, Mass. . . . xii
Symonds & Stevens, 70 State St., Boston . . . iv
Thomas Machine Company, 27 Cambria St., Boston . . . vii

PAGE
Thompson & Lichtner, The, Co., Inc., Statler Building, Boston iii
Thorpe, Lewis D., 6 Beacon St., Boston iii
Tomasello, A. G., & Son, 250 Stuart St., Boston vii
Travers-Sandell, Inc., 238 Main St., Cambridge viii
Tredennick-Billings Co., 10 High St., Boston xi
Trimount Dredging Company, 10 State St., Boston v
Turner, Howard M., 12 Pearl St., Boston iv
Warren Foundry and Pipe Co., 75 Federal St., Boston xi
Waterproofing Co., The, 65 Albany St., Boston v
Weston & Sampson, 14 Beacon St., Boston iii
Whidden-Beekman Engineering Co., 100 Arlington St., Boston iv
Whitman & Howard, 89 Broad St., Boston iii
Wolf, Ezekiel, 160 State St., Boston iv
Worcester, J. R., & Co., 79 Milk St., Boston iii
Wright & Potter Printing Co., 32 Derne St., Boston vii

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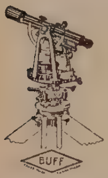

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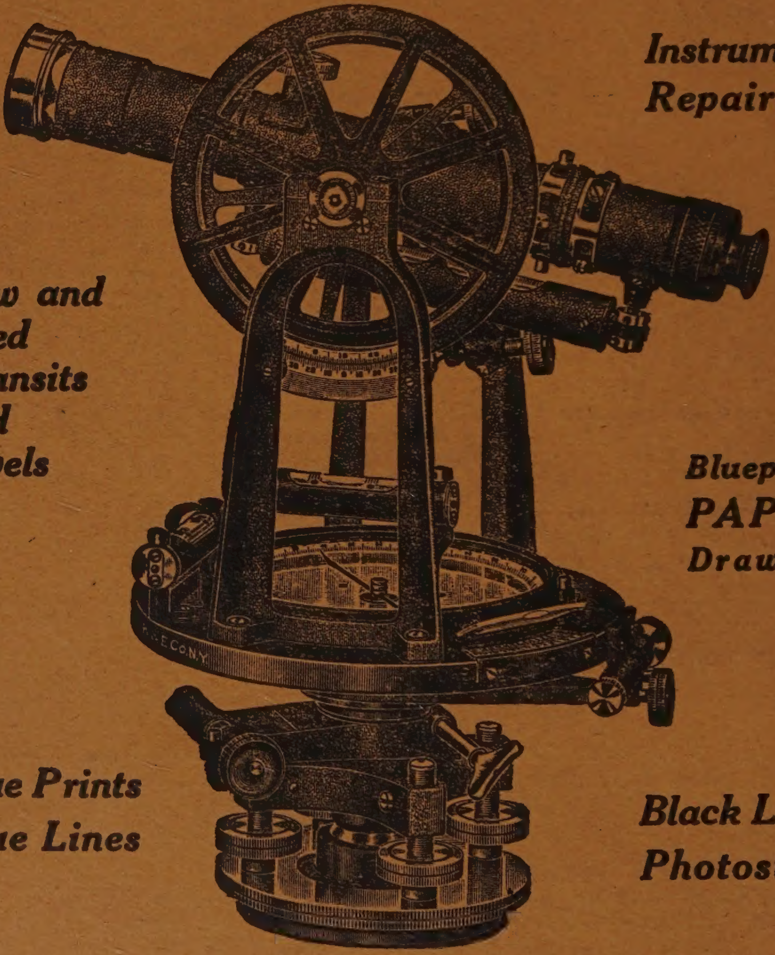

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